570

VOLUME 80 SEPARATE No. 570

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

DECEMBER, 1954

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

EFFECT OF SAMPLE DISTURBANCE ON THE STRENGTH OF A CLAY

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SOIL MECHANICS AND FOUNDATIONS DIVISION

{Discussion open until April 1, 1955}

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

EFFECT OF SAMPLE DISTURBANCE ON THE STRENGTH OF A CLAY

Max L. Calhoon,[1] J. M. ASCE

SYNOPSIS

Of the many variables that influence the laboratory test shearing strength of saturated homogeneous clays, the effect of sample disturbance (partial remolding) has received little constructive analysis in the past. It has generally been assumed that sample disturbance reduces the laboratory shearing strength and thereby results in an added factor of safety when the test results are applied to field design. It will be demonstrated herein, for certain clay soils of the type tested, that due to sample disturbance the actual field shear strength may be less than the laboratory test shear strength. A method of analyzing strength data is described which considers the effect of sample disturbance.

INTRODUCTION

Disturbances to the soil during sampling operations, during removal of the soil sample from the sampling tube, and during the process of trimming the specimen to size for a strength test cause a partial remolding of undisturbed soil specimens. In saturated clays which are sensitive to remolding, these disturbances cause a major deviation between observed laboratory strength and actual field strength. The effect of this variable frequently is not directly considered in analyzing laboratory strength results or in applying these results to the design of earth structures and foundations. Its effect is usually included in a factor of safety that is applied to the design and the designer hopes he has adequately cared for it. The fact that disturbances exist in all test specimens can lead to erroneous interpretations of triaxial shear test results which may be on the unsafe side in many saturated clays. A method of evaluating triaxial shear test results which considers the effect of sample disturbance is presented herein. This method is based on the results of studies made by the author at Northwestern University.

General Test Information

The following three definitions are significant as used in this paper. The term clay will refer to an extremely fine grained, highly plastic, highly impervious, homogeneous, cohesive, saturated soil. A sensitive clay is one that loses part of its shearing strength when in a partially or completely remolded condition. The sensitivity of a clay is defined as the ratio of its unconfined

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1. Soils Engr., Howard, Needles, Tammen and Bergendoff, New York, N. Y.

compressive strength at the natural water content in the undisturbed condition to its unconfined compressive strength at the natural water content in a completely remolded condition. The value of the sensitivity ratio for most clays is between two and four. For sensitive clays this ratio will be between four and eight. If the ratio is greater than eight then the clay is extra-sensitive. The compressive strength of a clay (difference between the major and minor principal stresses) is used to define the maximum applied axial stress which a triaxial compression test specimen will sustain before failing. Since a saturated clay has an effective internal friction angle equal to zero under conditions of no water content change during failure, the shearing strength of the clay is equal to one-half of the compressive strength.

The soil used in this study was obtained from Klug and Smith Company of Milwaukee, Wisconsin, in conjunction with an extensive foundation investigation at the Milwaukee Sewage Disposal Plant.(6)[2] The sample was taken in a five inch diameter thin walled tube in a sampler developed by Dr. J. O. Osterberg(3) of Northwestern University. The sample came from a depth of 38.5 feet to 42.3 feet below the ground surface and 29.5 feet to 33.3 feet below the Milwaukee datum (mean level of Lake Michigan).

The sample was part of an unusually uniform deposit of saturated clay with some organic matter and was highly sensitive to remolding and to sample disturbance. Its sensitivity ratio was in the vicinity of eight. The remolded unconfined compressive strength at the natural water content was difficult to obtain accurately since a remolded specimen would scarcely stand under its own weight. The soil appeared to be normally consolidated (never consolidated to any pressure greater than the existing effective overburden pressure) to a pressure of 1.1 tons per square foot as determined from the results of a one-dimensional consolidation test. This clay was a sedimentary deposit and was probably formed in a paludal environment of one of the ancient glacial great lakes. Its liquid limit was 71%, its plasticity index was 30, and its specific gravity was 2.67. Its average natural dry density was 59.2 pounds per cubic foot and its average natural water content was 67.7%. Its relative water content was 89% (also known as liquidity index or water-plasticity ratio). The uniform compressible stratum from which this soil came is approximately 35 feet in thickness. Natural moisture content tests and unconfined compression tests on undisturbed specimens throughout the depth of the stratum revealed no consistent trends toward lower moisture content or increased compressive strength as depth increased. This may indicate that this clay is of the type described by Terzaghi(9) as having a quicksand type of structure.

Method of Presenting Test Data

The commonly used method of evaluating test shear strength data is through the use of a Mohr diagram. Anyone familiar with such diagrams for clays knows that the simplified straight-line concept proposed by Coulomb is usually not observed in these soils. Many times there are erratic test results due to minor sample variations that are almost impossible to smooth intelligently on such diagrams. Therefore, a hypothesis proposed by Dr. P. C. Rutledge(5) for interpretation of results has been used.

Before describing this method, several types of shear tests for saturated clays should be reviewed. The first type is the quick triaxial test (Q test). In this test the specimen is placed in the shear device at its natural water content. A lateral pressure is applied but no water is allowed to drain from

2. Numbers in parentheses refer to references in the bibliography.

the specimen. An axial load is then rapidly applied until the specimen fails. It has been confirmed through many years of research that a given clay at constant water content will exhibit the same value of compressive strength no matter what the magnitude of the lateral pressure may be. If the lateral pressure is zero, then it is an unconfined compression test.

In a consolidated-quick triaxial test (Qc test) water is allowed to drain from the specimen under the influence of the lateral pressure. After drainage is essentially complete, the specimen is sealed against further drainage and the axial load is rapidly applied. After the specimen is sealed against drainage, the lateral pressure can be changed to any magnitude and the specimen will still show the same compressive strength, similar to a quick test. Since the water content at failure is reduced from the natural water content, the void ratio is also proportionately decreased. Therefore, the compressive strength is increased over that of the quick test. It should be pointed out that this increase in strength is not due to frictional properties and that the effective Qc friction angle is equal to zero.

In a slow triaxial test (S test) the specimen is allowed to drain under the lateral pressure similar to the Qc test. When this drainage is essentially complete the axial load is applied without sealing the specimen against drainage. In this test the axial load is applied slowly so that essentially complete drainage is achieved under each increment of axial load. Tests of this type generally show higher shear strength than the Qc test.

Rutledge's hypothesis for evaluating shear data will now be explained. There are two curves involved (see Figure 1). Curve A is a water content (or void ratio) versus logarithm of pressure curve which is unique in that it can be defined by either one-dimensional or triaxial consolidation. This implies that consolidation is independent of the minor principal stress. Curve B is a water content (or void ratio) versus logarithm of compressive strength curve which is unique in that it can be defined by unconfined compression tests, consolidated-quick or slow triaxial tests. Curve B is independent of pore water pressure. Curves A and B are essentially parallel when both are plotted on semi-logarithmic paper.

The unconfined compression or the quick triaxial compressive strength is the strength shown by Curve B at the natural water content, Wn (see Figure 2).

The consolidated-quick test strength is the compressive strength shown on Curve B at the water content at which the specimen failed. This water content is obtained from Curve A and corresponds to the lateral pressure used in the test (see Figure 2).

The slow triaxial test strength is the compressive strength which when added to the test lateral pressure is equal to the pressure on Curve A at the same water content as the compressive strength. There is only one such combination of values for each lateral pressure selected (see Figure 2 for a graphical illustration of determining the slow test strength).

It can therefore be seen that even though Curve B is defined entirely by unconfined compression tests (moisture content lowered by slow drying in a humid room) that the Q, Qc, and S triaxial strengths can be determined without performing these tests.

The use of this method of analyzing test data allows the designer to intelligently smooth erratic data and also gives valuable information as to possible causes of erratic data.

Tests Used

The tests used in this investigation were as follows: nineteen Qc triaxial

tests on undisturbed soil, eight Qc triaxial tests on remolded soil, one one-dimensional consolidation test with a 1.5 inch thick specimen on undisturbed soil, two one-dimensional consolidation tests with 0.75 inch thick specimens on undisturbed soil, and one one-dimensional consolidation test with a 1.5 inch thick specimen on remolded soil. The 0.75 inch thick one-dimensional consolidation specimens were remolded at the completion of consolidation and brought back to their natural moisture contents, then reconsolidated. All one-dimensional consolidation test specimens had an area of 100 square centimeters. The average size of the triaxial test specimens was 3.6 cm. in diameter and 6.0 cm. in height. The results of all of these tests are shown in Figures 3 and 4.

It can be seen that the undisturbed compressive strength results are not as uniform as desirable but curves representing various initial water contents could be drawn with fair accuracy. The non-uniform behavior was probably due to slight variations in the initial condition of the soil from one specimen to another.

Analysis of Test Results

Validity of Rutledge's Hypothesis

The undisturbed and remolded compressive strength curves representing an initial water content of 67% have been selected for analysis. It can be seen in Figure 4 that the average undisturbed triaxial consolidation curve does not coincide with the 1.5 inch thick one-dimensional consolidation curve for undisturbed soil. It can be seen in Figure 5 that the compressive strength curve for undisturbed soil is not parallel to the 1.5 inch thick undisturbed one-dimensional consolidation curve. At first, one might think that this is in direct opposition with Rutledge's hypothesis. Therefore, further analysis is required to determine the reasons for these apparent discrepancies.

In looking again at Figure 4 the reader will note that the consolidation curves for the 0.75 inch thick one-dimensional consolidation specimens of undisturbed soil are displaced downward from the curve of the 1.5 inch thick consolidation specimen. It is known that the consolidation curve for remolded clay is displaced downward from the consolidation curve for undisturbed clay. Van Zelst[11] has shown for undisturbed saturated clays, for a given specimen diameter, that as the specimen thickness decreases the consolidation curves are displaced further downward toward the consolidation curve for completely remolded soil. He concluded that this vertical displacement was due mainly to sample disturbance in the specimens and was inversely proportional to the initial specimen volume. Therefore, as the initial specimen size decreases, the remolding and downward displacement of the consolidation curves increase. The percentage of remolding in a 0.75 inch thick specimen would then be twice that in a 1.5 inch thick specimen and the remolding in a 0.375 inch thick specimen would be four times that in a 1.5 inch thick specimen. If the 1.5 inch thick specimen is 10% remolded, then the 0.75 inch specimen is 20% remolded and the 0.375 inch specimen is 40% remolded. Van Zelst used this phenomenon to devise a method of extrapolating the position of a consolidation curve representing soil with no remolding, or in other words, a curve representing natural soil in the field. The procedure for doing this is described later.

The position of consolidation curves for saturated clays is, therefore, a function of the initial volume of the test specimen. This explains why the curves for the undisturbed 0.75 inch and 1.5 inch thick one-dimensional consolidation specimens do not coincide. It is logical to extend this idea to the triaxial

consolidation curve for undisturbed soil. The volume of the triaxial shear test specimens was much less than the volume of the 1.5 inch thick one-dimensional consolidation specimens. It could not be expected, therefore, that the consolidation curve for the triaxial test specimens would coincide with that of the 1.5 inch thick one-dimensional consolidation test specimen.

In Figure 5 it can be seen that the compressive strength curve for undisturbed soil is parallel to the average triaxial consolidation curve for undisturbed soil. The compressive strength curve is therefore parallel to the consolidation curve that represents the same amount of remolding as the strength curve represents.

The above facts suggest that Rutledge's hypothesis should be modified for soils that are sensitive to remolding as follows:

1. Curve A is unique in that it can be defined by either triaxial or one-dimensional consolidation if the triaxial and one-dimensional consolidation specimens are disturbed to the same degree.
2. Curve B is unique if all test specimens are disturbed to the same degree.
3. Curves A and B are parallel only if they both represent soil specimens which were disturbed to the same degree.

Method of Obtaining Field Strength

Since it has been fairly definitely shown that the degree of remolding affects the position of the consolidation curves, it is logical to surmise that the degree of remolding would also affect the position of the compressive strength curves. As can be seen in Figure 5, the remolded compressive strength curve is displaced downward from the undisturbed compressive strength curve. If the amount of disturbance in the triaxial shear test specimens on undisturbed soil were known, it would then be possible to calculate the position of a compressive strength curve representing soil with no disturbance. Assuming that the percentage of disturbance in the undisturbed triaxial specimens is known, say 20% for this example, the calculation would be as follows. The vertical distance (at any value of compressive strength) between the undisturbed strength curve and the 100% remolded strength curve is equal to 80% or 8 units. The curve representing zero remolding is then 2 units above the undisturbed curve. The operation for determining the position of the strength curve representing no remolding may be carried out either mathematically by reference to the vertical scale on the graph or it may be accomplished graphically through the use of a scale and dividers. Several points on the strength curve for zero disturbance should be located and then connected with a smooth curve.

The percentage of remolding in the undisturbed triaxial specimens could be calculated if the position of a consolidation curve representing zero remolding were known by noting the position in the vertical direction of the average triaxial consolidation curve for undisturbed soil with respect to the 100% remolded consolidation curve and the consolidation curve for zero remolding. This calculation would be similar to that described above. The vertical distance between the curves for zero percent and 100 percent remolding represents 10 units (at any value of pressure). If the average triaxial consolidation curve for undisturbed soil is two units below the consolidation curve for zero remolding, then the triaxial test specimens would be 20% remolded, etc.

The first problem, then, was to obtain a consolidation curve representing natural soil in the field which is not disturbed in any way (zero percent remolded). To do this, the method proposed by Van Zelst[11] was utilized. This method will now be described.

It is assumed that the operation of trimming undisturbed one-dimensional consolidation specimens disturbs the specimens to the same depth no matter what the thickness of the consolidation specimen may be. This is merely an analogy to express the idea that a 0.75 inch thick specimen is remolded twice as much as a 1.5 inch thick specimen of the same diameter, etc. This is a reasonable assumption since the trimming operation for all specimen sizes is the same.

The tests that are required are one undisturbed one-dimensional consolidation test using a thick specimen (say 1.5 inch) and one test using a thin specimen (say 0.75 inch). Also, a completely remolded one-dimensional consolidation test is needed using either of the previously mentioned specimen sizes.

The moisture content versus log pressure diagrams are plotted for each of the above three tests. A percentage of remolding (P_1) is assumed for the thick specimen (thickness = T_1). The percentage of remolding (P_2) for the thin specimen (thickness = T_2) is equal to (T_1/T_2) (P_1). (This is true only if the areas of both specimens are equal.) The percentage of remolding for the remolded specimen is, of course, 100%. The three consolidation curves are then examined to determine whether the assumed percentages of remolding (P_1 and P_2) place the three consolidation curves correctly in the vertical direction with respect to each other. For example, if P_1 is assumed as 10% for a 1.5 inch thick undisturbed specimen then P_2 is 20% for a 0.75 inch thick undisturbed specimen. The vertical distance between the 100% remolded curve and the curve for the 1.5 inch thick specimen is 90% or 9 units (at any value of pressure). The vertical distance between the 1.5 inch and 0.75 inch curves should be one unit and the vertical distance between the 0.75 inch and the 100% remolded curve should be 8 units. The distance is then measured on the graph of the three curves and if they are actually spaced vertically with respect to each other so that the above percentages hold true, then the assumed P_1 and P_2 values are correct. P_1 and P_2 are found by trial and error. After the correct values for P_1 and P_2 are found then a consolidation curve representing zero remolding can be easily extrapolated. If P_1 and P_2 are found to actually be equal to 10% and 20% respectively, then the curve representing zero remolding would be one unit above the curve for the 1.5 inch thick specimen.

Figure 6 shows the above mentioned average test curves plus the calculated consolidation curve for a natural soil in the field. It is interesting to note that this calculated curve at the initial water content lies within the probable range of preconsolidation pressures for the undisturbed 1.5 inch thick consolidation specimen and is therefore considered valid.

The positions of the consolidation curves representing 0% and 100% remolding are now known. It is a simple matter to compute the percentage of remolding in the undisturbed triaxial shear specimens by noting the position of the average triaxial consolidation curve in the vertical direction relative to the 0% and 100% remolded consolidation curves as shown previously.

It is now known what percentages of remolding the remolded and undisturbed compressive strength curves represent. A simple extrapolation of distances in the vertical direction will give the position of a compressive strength curve representing 0% remolding, or in other words, representing the compressive strength of natural soil in the field. An example of this extrapolation has been given previously.

Figure 7 shows the laboratory (triaxial) and field (calculated) consolidation curves and compressive strength curves. The average triaxial consolidation curve is used for the laboratory Curve A since it and the laboratory compressive strength curve both represent the same degree of disturbance.

Figure 8 shows consolidated-quick shearing strength envelopes which are drawn from Figure 7. The magnitude of the ordinate to the field curve at zero normal pressure is not known since it is not known whether the field consolidation curve progresses upward above the natural water content as a straight line to zero pressure or whether it bends to the left somewhat above the liquid limit as suggested by Terzaghi[10]. If the field consolidation curve is a straight line to zero pressure and the compressive strength curve is parallel to it, then the shearing strength of the soil is zero at zero normal pressure. This implies that "cohesion" is a function of the pressure history of the soil. It is readily seen that the field shearing strength curve is below the laboratory shearing strength curve at all values of normal pressure. This phenomenon is not necessarily confined to highly sensitive clays. It is entirely possible for clays of low and medium sensitivity to exhibit lower shearing strength in the field than in the laboratory.

Discussion of Results

An analogy can be made wherein the undisturbed triaxial specimens are considered to consist of a core of soil which is completely undisturbed and is surrounded by an annulus of completely remolded soil. The average consolidation curve of this specimen would necessarily fall somewhere between a curve representing no disturbance and a curve representing 100% remolding. The undisturbed core would follow the same consolidation curve as completely undisturbed soil and the remolded annulus would follow the same consolidation curve as completely remolded soil. Since the remolded soil exhibits a higher compressive strength than the undisturbed soil for any given value of test lateral pressure, it would be expected that the average compressive strength of this specimen would be greater than the compressive strength of a specimen of completely undisturbed soil.

It should be noted in Figure 8 that the natural shearing strength in the field is the ordinate to the field curve at the preconsolidation pressure which was determined from the calculated field consolidation curve. This shearing strength is greater than the laboratory unconfined test shearing strength (ordinate to the laboratory shearing strength curve at zero normal pressure).

One might logically wonder, then, since the natural field shearing strength is greater than the laboratory unconfined test shearing strength, why failures occur in the field when the design was based on the laboratory unconfined test shearing strength. The reader is reminded that the shearing strength of clays is a very complex phenomenon and is a function of many variables. One of these variables which appears to be important is the rate of application of the axial load to the specimens in the laboratory as compared to the rate of loading in the field during construction. A fairly complete discussion of this subject can be found in reference 2. Some other factors that affect the strength and stability of clays which the designer may not consider are planes of weakness in the clay; progressive failure in the field; short term fluctuations in ground water pressures, possibly as short as an hour; test specimens which are too small to be representative of the average condition of the soil; incorrect assumptions as to the stress conditions in the field prior to failure. It is evident that failure can be the result of a multitude of causes other than using an incorrect value for the shearing resistance of the soil.

It should also be pointed out that the shearing resistance shown on the laboratory curve of Figure 8 at the preconsolidation pressure is somewhat on the unsafe side and should not be used. Many designs have been based on this value of shearing resistance. In some cases it may be over-conservative to

use the unconfined compression test results also. Therefore, a careful study should be made of the shearing resistance considering all factors before selecting a value to be used for design.

CONCLUSIONS AND RECOMMENDATIONS

On the basis of the studies made it has been found for a sensitive saturated clay that the actual shearing strength of this soil in the field would be less than the laboratory test shearing strength at any given value of normal pressure.

A recommended testing program and method of analysis to determine shearing resistance for clays of the general type used in this investigation is as follows:

1. Samples should be carefully obtained by means of hand cutting or by the use of large diameter sampling tubes (at least four inches in diameter if possible).
2. Laboratory testing should include the following tests:
   a. One undisturbed one-dimensional consolidation test using a thick specimen - say 1.5 inches.
   b. One undisturbed one-dimensional consolidation test using a thin specimen - say 0.75 inch.
   c. One remolded one-dimensional consolidation test using either of the specimen sizes of a or b above.
   d. Two or three unconfined compression tests on undisturbed soil at the natural water content.
   e. Two or three unconfined compression tests on remolded soil at the natural water content to determine the sensitivity ratio.
   f. At least four consolidated-quick triaxial shear tests on undisturbed soil, each at different lateral pressures. If the natural water content of the specimens varies by more than 1%, enough specimens should be used to establish compressive strength curves for a range of initial water contents (if the soil has a tendency to show different strength curves for different initial water contents). The triaxial consolidation characteristics should be carefully observed so that the triaxial consolidation curve can be constructed.
   g. At least four consolidated-quick triaxial shear tests on remolded soil, each at different lateral pressures.

The procedure for analyzing the data based on the methods described in this paper would be as follows:

1. Extrapolate field virgin consolidation curve from the tests in a, b, and c above.
2. Plot undisturbed triaxial consolidation curve and undisturbed compressive strength curve.
3. Plot remolded consolidation curve (may be either one-dimensional or triaxial or average of both depending on the validity of the data) and remolded compressive strength curve.
4. Determine percentage of remolding in undisturbed triaxial specimens.
5. Extrapolate field compressive strength curve for the average natural water content expected in the field.
6. Plot shear strength envelopes for the laboratory and field similar to Figure 8 using the triaxial consolidation curve as the laboratory Curve A (both shear envelopes should represent the same initial water content).
7. Determine the shearing strength to use in the design from an analysis of the above curves.

This proposed testing program and method of analysis will give as complete a picture of the shearing strength characteristics of saturated, sensitive clays as it is possible to obtain at our present state of knowledge of the subject.

ACKNOWLEDGMENTS

The writer wishes to express appreciation to Dr. P. C. Rutledge for the conception of the initial idea for this investigation, for his many helpful suggestions during the early part of the testing, and for his critical review of the manuscript for this paper; to Dr. J. O. Osterberg for his suggestions and criticisms throughout the entire investigation; and to John W. N. Fead and Elmer A. Richards for their constructive criticisms and assistance in performing many of the laboratory tests.

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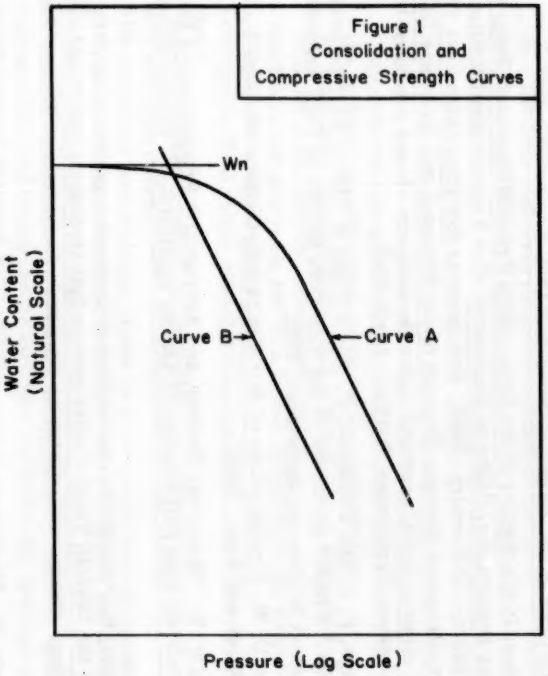


Figure 1
Consolidation and Compressive Strength Curves

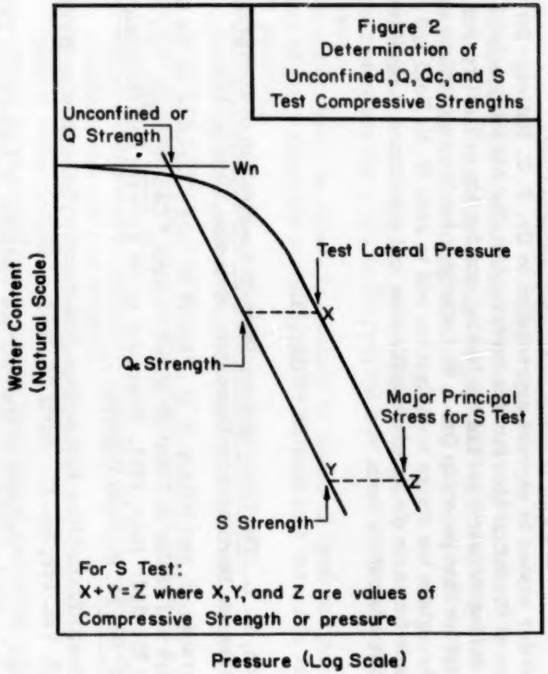


Figure 2
Determination of Unconfined, Q, Qc, and S Test Compressive Strengths

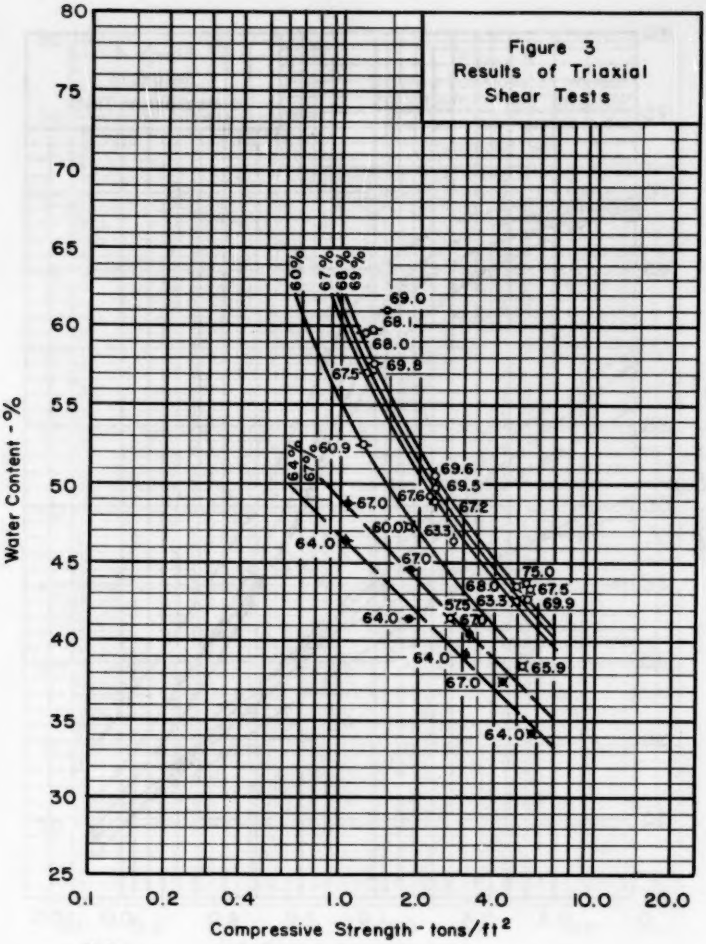


Figure 3
Results of Triaxial Shear Tests

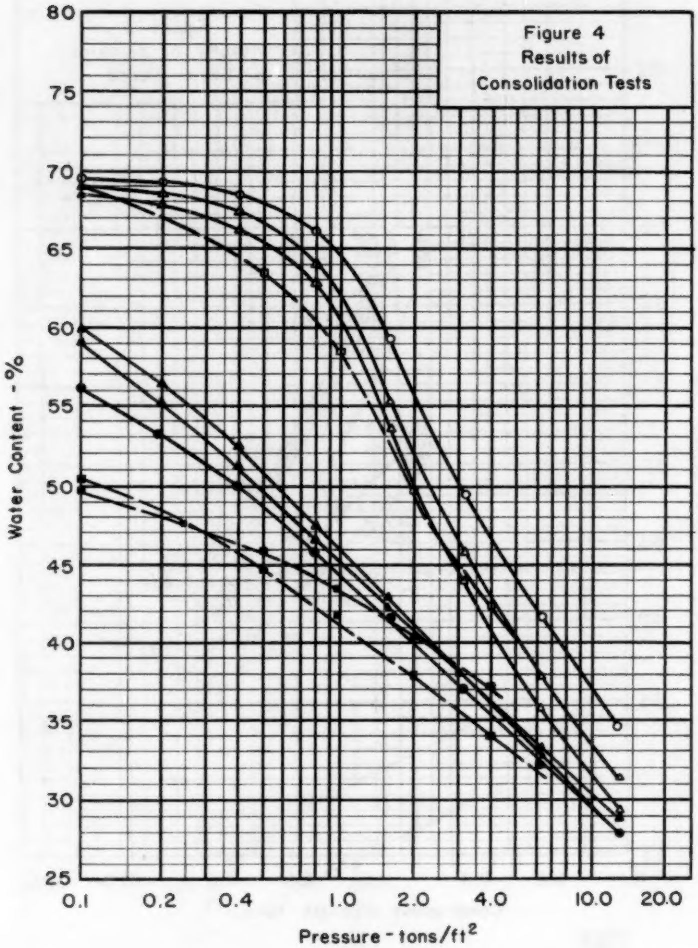
Notes:

Test Lateral Pressure - tons/ft²	0.5	1.0	2.0	4.0
Undisturbed Soil		-o-	♀	¤
Remolded Soil	✦	-●-	♦	✖

—— Undisturbed Compressive Strength Curves for initial water contents of 60%, 67%, 68% and 69%

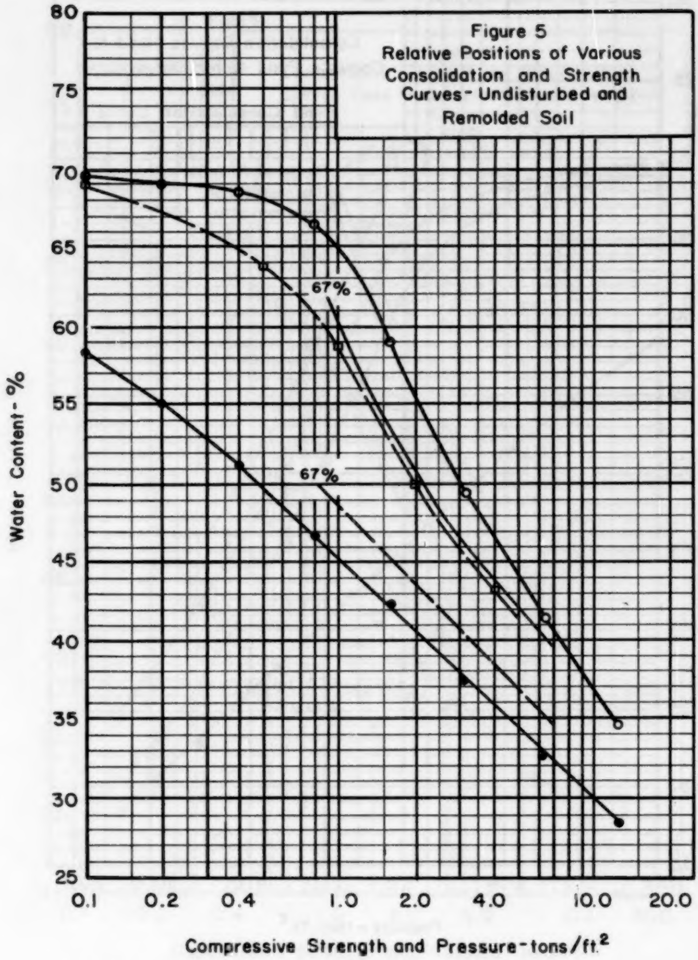
— — Remolded Compressive Strength Curves for initial water contents of 64% and 67%

All numbers indicate water content.

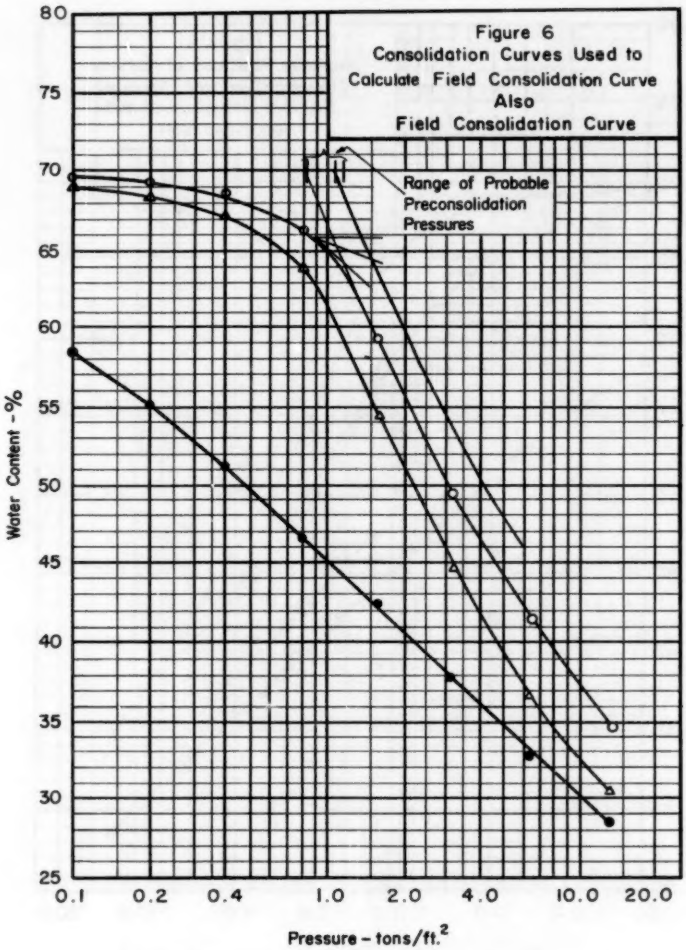


Notes:

- ○ Undisturbed One-Dimensional 1½" thick
- △ Undisturbed One-Dimensional ¾" thick
- □ Undisturbed Triaxial - Average of 17 tests
- ● Remolded One-Dimensional 1½" thick
- ▲ Remolded One-Dimensional ¾" thick
- ■ Remolded Triaxial

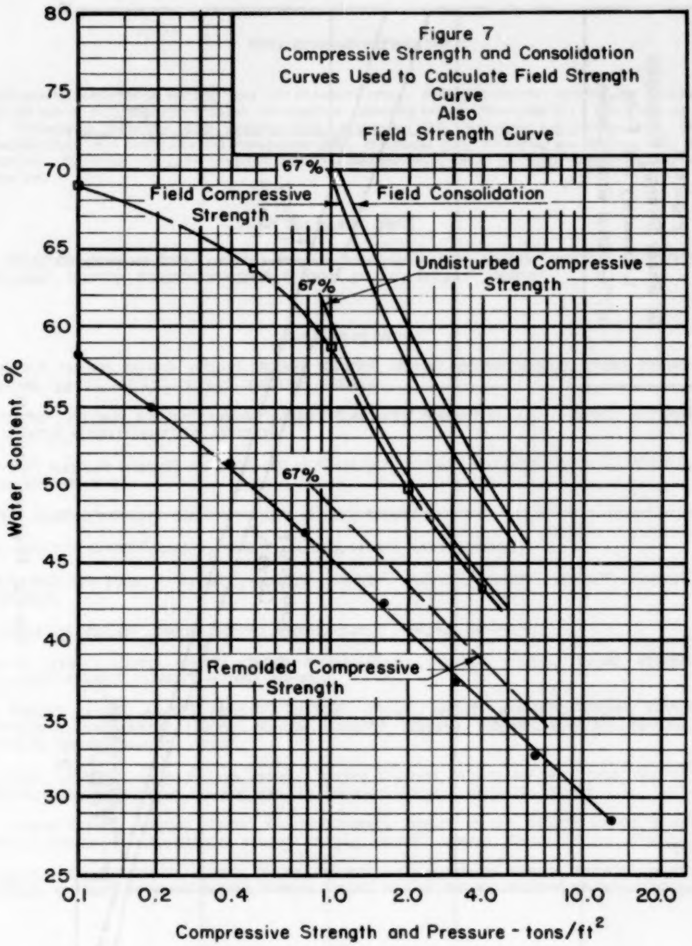


Notes:
- ○ Undisturbed One-Dimensional 1½" thick
- □ Undisturbed Triaxial - Average of 17 tests
- ● Remolded One-Dimensional - Average of 3 tests
- ——— Undisturbed Compressive Strength Curve
- — — Remolded Compressive Strength Curve



Notes:

- o Undisturbed One-Dimensional 1½" thick
- △ Undisturbed One-Dimensional ¾" thick-Average of 2 tests
- ● Remolded One-Dimensional-Average of 3 tests
- —— Field Consolidation Curve (Calculated)



Notes:

Consolidation Curves:

□ Undisturbed Triaxial - Average of 17 tests

● Remolded One - Dimensional - Average of 3 tests

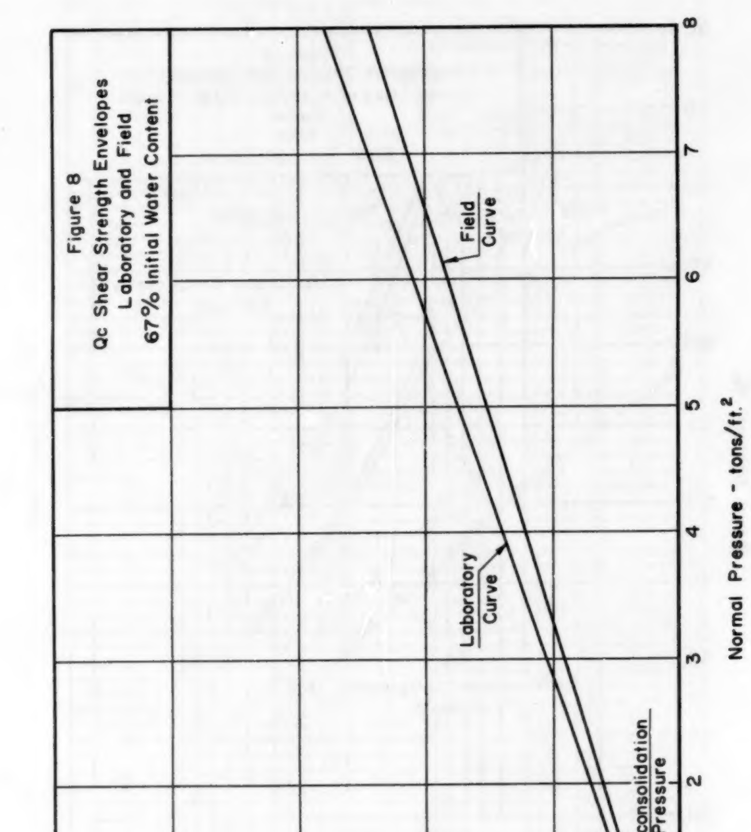


Figure 8
Qc Shear Strength Envelopes
Laboratory and Field
67% Initial Water Content

PROCEEDINGS-SEPARATES

The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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APRIL: 428(HY)[c], 429(EM)[c], 430(ST), 431(HY), 432(HY), 433(HY), 434(ST).

MAY: 435(SM), 436(CP)[c], 437(HY)[c], 438(HY), 439(HY), 440(ST), 441(ST), 442(SA), 443(SA).

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c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.